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MINIMUM FOOTING DIMENSIONS FOR A GIVEN SETTLEMENT IN GRANULAR DEPOSITS

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ABSTRACT

The allowable pressure applied to the foundation in granular soils governed by the consideration of settlement rather than shear strength of soil except for very narrow, shallow footings on loose materials with high water table. Accurate prediction of the settlement of structures founded on granular material is of considerable importance. Several methods are available for predicting the settlement of footing on granular deposits. This paper presents the comparison of the three methods namely, DeBeers and Martens (1957) using 2:1 pressure distribution, DeBeers and Martens (1957) using Bousinessiq stress distribution charts and Schmertmann (1970) Method for proportioning three shapes of foundations namely circular, square and rectangular for equal settlement on granular deposits using field test results.

Key words: Granular soil, foundation settlement, footing depth etc.

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1. INTRODUCTION

Calculation of foundation settlement is a basic and fundamental component of foundation engineering and is a common procedure performed by practicing geotechnical engineers. The deformation behavior of shallow foundations deriving their support from primarily granular

particulate soil deposits such as sands and gravels largely controls the final design of structures resting on these materials. This is largely due to the fact that the limit equilibrium behavior, i.e., the bearing capacity, of shallow foundations resting on granular deposits is typically of such a large magnitude, that the allowable settlement criteria established by the engineer will control the overall design.

In transportation related construction, one of the most common uses of shallow foundations is in the support of bridge structures, especially in dry crossing situations, where highway over passes are needed for crossing over other highways, railroads or other structures. Provided that settlements can be accurately estimated, a shallow foundation provides a more economical foundation than either driven or drilled deep foundations.

2. LITERATURE REVIEW

Bozozuk (1978) presented the results of a performance survey of existing bridges in the U.S. and Canada to determine the movement that could be tolerated by a structure. Based on the results of the performance of about 120 abutments and piers on spread footings, the vertical movements ranged from approximately 0 to 1000 mm, while the horizontal movements ranged from 0 to 150mm.Moulton (1986) surveyed a large number of existing highway bridges in the U.S. and showed that generally, most bridges can tolerate more than 25 mm of settlement, that bridges founded on spread footings do not settle more than bridges on piles and that damage to bridges cannot be attributed to spread footing foundations more than to pile foundations. The tolerable movement of bridges and the use of shallow foundations have also been discussed by Wahls (1983) and Yokel (1990) .A numbers of studies have been published in the past 40 years comparing the results of calculated settlements with observed settlement of shallow foundations on granular soils. Some of these studies have been related to proposing a new settlement prediction method, while others have attempted to provide a comparison among various methods to evaluate whether or not any one particular method appears to provide superior accuracy over another. Most notably, review papers which summarize settlement observations or provide comparisons between predicted and observed settlements have been presented by Alpan (1964); Schmertmann (1970); Jorden (1977); Arnold(1980); Burland and Burbidge (1985); Jeyapalan and Boehm (1986); Maail (1987); and Berardiand Lancellotta (1991).The reliability of settlement estimates for shallow foundations on granular soils has also received considerable attention and has been discussed by Schultze and Sievering (1977); Tan and Duncan (1991); Nova and Montrasio (1991a, 1991b); Cherubini and Greco (1991); and Berardi and Lancellotta (1994).

There are essentially two separate design approaches that can be taken when using the results of in situ tests in geotechnical design: (1) Indirect Design and (2) Direct Design. In general, most engineers are currently using an indirect approach to apply the results of in situ tests to specific problems, although there are a number of cases in which direct design may be useful and appropriate. In the indirect design method, one of the most common ways that in situ tests can be used in the design problem is to estimate elastic modulus of the soil and then calculate settlement for generalized elastic equation. Other, more direct design approaches based largely on empirical observations, for predicting settlements using the results obtained from a variety of in situ field tests namely. Standard Penetration Test (SPT), Cone Penetration Test (CPT), Pressure Meter Test (PMT), Dilatometer Test (DMT), Plate Load Test (PLT), Drive Cone Test (DCT). In the present study the cone penetration test (CPT) has been used to estimate settlement of shallow foundations on granular soils. Approach is based on the results of the CPT rely on the tip resistance values obtained from the test. Methods that are used to find minimum foundation dimension for a given settlement in the present study

are De Beer and Martens (1957) using 2:1 pressure distribution, De Beer and Martens (1957) using Bousinesq stress distribution charts and Schmertmann (1970) Method.

2.1. DeBeer and Martens (1957) Method

DeBeer and Martens (1957) is based on the semi-empirical Terzaghi-Buisman formula for calculating settlements:

$$s = (2.3/C)\log[\frac{(p_0^{'} + \Delta p^{'})}{p_0^{'}}]H$$

Where: s = settlement, C =constant of compressibility, $p_0^{'}$ = effective overburden stress, $\Delta p^{'}$ =increment of stress at depth due to the footing stress, H = thickness of the layer.

Settlement from individual soil layers can be calculated and then summarized to have the total settlement for all layers. The constant of compressibility, C is obtained from the CPT tip resistance, q_c as:

$$C = 1.5\frac{q_c}{p_0^{'}}$$

The method was intended to provide a "safe upper limit of settlements" and compared with the settlements of several bridge abutments and piers generally gave estimates of about 2 times the observed settlement. Because of over estimation the less conservative relationship suggested by Meyerhof (1965) has been more widely used is given by

$$C = 1.9\frac{q_c}{p_0^{'}}$$

GargaQuin (1974) have taken compressibility by the following relation, and the method which uses this relation is called modified Buisman DeBeer.

$$C = 2.9\frac{q_c}{p_0^{'}}.$$

The value of $\Delta p^{'}$ is calculated from 2:1 method (Bowels (1984)) and also from Bousinesq stress distribution charts. The maximum depth of the zone of influence may be taken as the depth below the foundation to which the change in vertical effective stress equals 10% of the applied surface stress.

If the cone tip resistance q_c is constant with depth, it is suggested to use the Boussinesq equation to determine $\Delta p^{'}$ as:

$$\Delta p^{'} = \frac{3q_c\cos^5\theta}{2\pi Z^2}$$

This method is only applicable to normally consolidated sands.

2.2. Schmertmann (1970) Method

Schmertmann (1970) proposed a method for calculating the settlement of shallow foundations on sands by subdividing the compressible zone beneath the footing into individual layers and then summing the settlement of each sub layer. The method relies heavily on an assumed vertical strain distribution which develops beneath the footing. As presented originally by Schmertmann (1970), this method is often referred to as the "2B- 0.6" method which described the approximate strain influence diagram proposed by Schmertmann to calculate

settlements over a zone of influence equal to 2B below the footing. The area of the strain influence diagram is related to the settlement. Settlement is calculated from the expression:

$$s = C_1 C_2 q_n \sum_0^{2B} \left(\frac{I_Z}{E_S}\right)\Delta Z$$

Where: s =settlement, q_n = net foundation base stress (total foundation stress- effective overburden stress at foundation level, $p_0^{'}$), I_Z = strain influence factor, E_S = soil modulus, ΔZ = thickness of elemental layer.

C_1 = depth correction factor $=1-0.5\dfrac{p_0^{'}}{q} \geq 0.5$,

C_2 = creep correction factor $=1.0+0.2\log\dfrac{t}{0.1}$, where t =time in years.

In order to obtain the strain influence factor, I_Z at the midpoint of each soil layer, it is necessary to construct the strain influence diagram. To construct the strain influence diagram for a particular case, the following approach is used:

For ax symmetric footings (square and round) I_Z = 0.1 at depth=0, I_Z =0 at depth=2B;

For plane strain footing (L/B > 1.0) I_Z =0.2 at depth=0, I_Z =0 at depth = 4B

The method of estimating settlement proposed by Schmertmann (1970) is primarily intended for use with cone penetration test data. The CPT has the obvious advantage of providing a near continuous record of penetration resistance, especially if an electric CPT is used and thus provides larger data base for delineating individual important sub layers within the compressible zone as well as allowing statistical averaging of data within a layer. Schmertmann (1970) suggested that based on screw plate tests, the soil modulus could be evaluated from: $E_S = 2q_c$

In this study methods pertaining to Duch cone test are used for the following reasons:

- IS: 8009(Part I) 1976 recommends DeBeer Martin (1957) method used cone penetrations only.
- Simons and Menzis (1977) mentioned that the Schmertmann (1970) method using static cone penetration values gives best agreement with the observed settlement on average.
- The advantage to using the CPT is to provide a more continuous sounding, however it is felt that this would only be achieved at considerable expense of equipment and manpower to the state and could actually create a slowdown in data reduction and interpretation for typical projects.

3. PROPORTIONING OF FOOTINGS FOR EQUAL SETTLEMENTS

In granular deposits the allowable settlement is usually exceeds before soil rupture conditions become significant. However, the recorded total settlement of footing on granular deposits is being in the range of 25mm to 50mm. Skempton and MacDonald (1955) has suggested a maximum settlement of 40mm. A footing of width of B placed on a homogeneous granular deposit at a depth of 1.5m, under submerged condition is considered for proportioning of footing for maximum settlement of 40mm. For a given cone penetration resistances by changing the load on the footing is studied, to have the comparison of footing sizes based on the three mentioned methods. Increase in vertical stress under the center of uniformly loaded flexible footing for different shapes can be calculated using Janbu, Bjerrum and Kjaernsli (1956) design charts. Assuming pressure bulb is assumed to extend to depth (Z) equal to

depth of layer (H). This Z value differs for different shapes, for Circular Footing Z equals to 2.8B, for Square Footing Z equals to 3.0B, and for Rectangular Footing when α (L/B) = 2.0then Z equals to4.5B, α=3.5 then Z equals to 5.8B, α =5then Z equals to7.0B.

Footingsizes of various shapes(circular, square, rectangular) are determined for loads 500Kn/m^2,1000Kn/m^2,1500kN/m^2,2000kN/m2 2500kN/m^2 and for various cone penetration resistance values ranging from 1000Kn/m^2 to3000Kn/m^{2}using DeBeer and Martens (1957) Method and Schmertmann (1970) Method. DeBeer and Martens (1957) Method is used twice for determination of footing sizes for various shapes considering the pressure distribution by 2:1 method (Bowels (1984)) and by using Bousinesq stress distribution charts.

4. RESULTS AND DISCUSSION

Load verses Area of Footing graphs are drawn for (DeBeer and Martens (1957) using 2:1 pressure distribution, (DeBeer and Martens (1957) using Bousinesq stress distribution charts and using Schmertmann (1970) Method by taking cone penetration resistance equals to 1000kN/m^2 as shown in the fig. 1, fig. 2 and fig. 3. respectively.

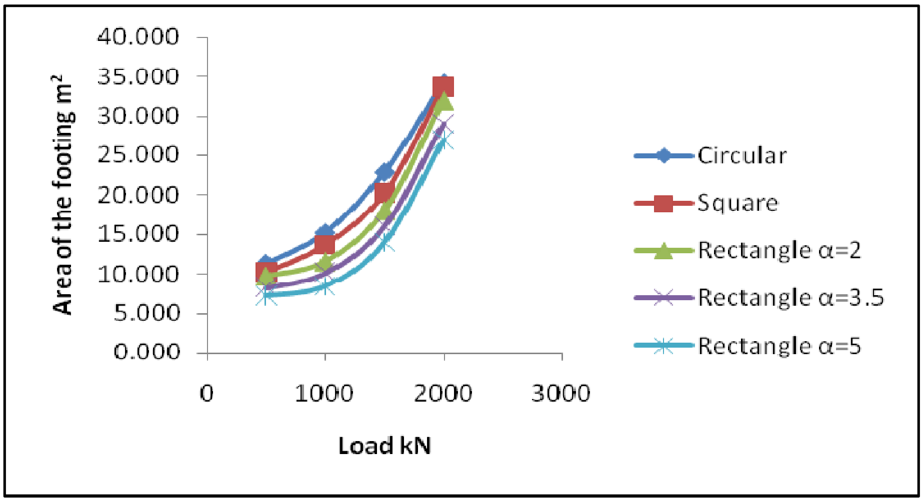

Figure 1 Load verses Area of Footing graphical representation for(DeBeer and Martens (1957) using 2:1 pressure distribution

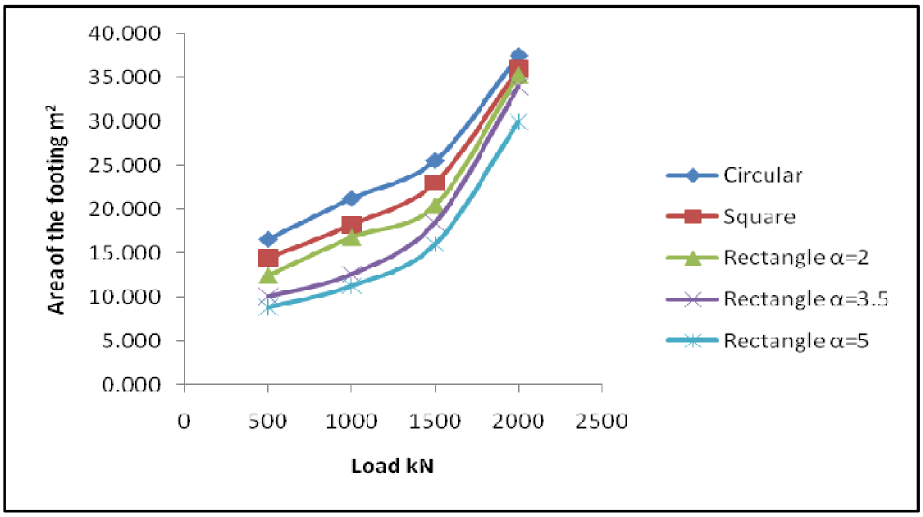

Figure 2 Load verses Area of Footing graphical representation for(DeBeer and Martens (1957) using Businessq stress distribution charts

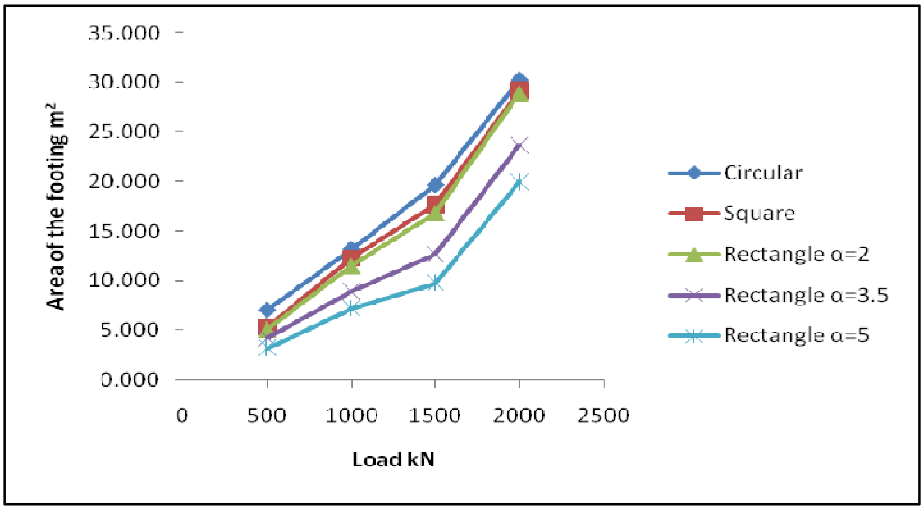

Figure 3.Load verses Area of Footing graphical representation for Schmertmann (1970) Method

It is observed that when all parameters are same, a circular footing occupies more area than square or rectangular footing. For a given load with different cone penetration resistance also studied the area of footing decreases with increase of cone penetration resistance. Footing design based on Schmertmann (1970) Method analysis is more economical than the (DeBeer and Martens (1957) using 2:1 pressure distribution, (DeBeer and Martens (1957) using Bousinesq stress distribution charts.

5. CONCLUSION

Design of footing based on shear strength criteria would result the same contact area whatever be the shape of the footing. Designing of footing based on settlement theory would result different contact areas for different shapes of footings. It is inevitable to arrive economical proportioning of footing which uses least contact area. Thus rectangular footing is the most economical shape than circular, square for a shallow footing on a granular deposit.

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